STABILIZATION OF LOESS BY CEMENT FOR FOUNDATION PURPOSES

by

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INTRODUCTION

Statement of the Problem

Vast areas of eastern Europe, southern and central Asia, and of the midwestern United States are covered with fine-grained, wind-deposited, soil called loess. The depth of this loess varies from a few inches to well over a hundred feet. All soils of this type are characteristically subject to considerable settlement and to loss of shear strength with increased water content. This gives rise to numerous foundation failures in all types of structural designs. To avoid failures piles are often used to transmit the loads to the underlying bed rock or hard soil stratum. Since pile foundations are relatively expensive, many structures located in loess soil areas are founded on spread footings in this soil and evidence of failure by either consolidation or shear is common in these structures.

Purpose of the Study

In view of the ever-growing construction activities in the loess mantled regions, the search for inexpensive foundation techniques has acquired considerable economical importance. The purpose of this study was to investigate the possibility of in-place stabilization of loess soils which would allow the use of spread footings as an economical solution of foundation design.

Scope of the Study

A laboratory study was conducted to determine the improvement in the engineering properties of loess soils when portland cement and sodium sulfate were used as additives. The relative improvement was measured in tests of compressive strength, wetting and drying, density, and plasticity.

The laboratory study was supplemented by a review of the existing literature as a guide to the selection of the quantities of additives to be used and of the types of tests which would best predict the improvement in field behavior of the treated soil.

LITERATURE REVIEW

Loess Soil

Physical Properties of Loess Soil

The most unique feature of loess is the grain-size distribution. Silt particles, 0.05 to 0.005 mm, are predominant, with the percentage reaching 70 to 90%. The moisture content of loess soils is extremely variable and ranges from a low of 3% to full saturation of some 30%. In the weathering zone low moisture contents of 7 - 10% are most common. Porosity is the most characteristic parameter of subsident loess soil and varies from 40 to 62% with the most common range being from 44 to 53%. Non-subsident varieties are usually of low porosity, that is less than 40%. (1)

The liquid limit of wind-blown silt is approximately 30, and the plasticity index is approximately 6. In the natural state, wind-blown silts are well drained because true loess deposits have a very high vertical permeability. Dry loess soils are difficult if not impossible to compact and wet loess exhibits "quick" properties in either cut or fill sections. (2) These materials are very sensitive to small changes in density and moisture content making them difficult to handle in both foundation and embankment work. (3)

The shearing resistance of silt and silty sand is similar to that for clean sand. The values of ϕ_s, obtained from the slow shear test, range from about 27° to 30° for the loose

state and from 30° to 35° for the dense state. (3)

Foundations on Loess Soil

The values of the standard penetration test N, vary from 4 to 20, but this does not appear to be a sufficiently reliable indication of characteristics of loess to serve as a basis for design because the uniform consistency and the high void ratio of loess facilitates penetration of the sampling tube. Performing a load test for the determination of allowable soil pressure is advisable in all cases for design. (3)

The subsident capacity of loess soils is reflected by vertical settlement due to the compaction by the wedge effect of thin water films. (4) Increased moisture content will cause the high porosity loess soils to settle greatly which will be detrimental to all types of structures located on them.

Drannikov (1) with tests of subsidence of settlement plates on loess soil due to loading and wetting showed that loess soil under a low pressure of 1.0 to 1.25 kg/sq.cm does not subside significantly when wetted, while at 2 to 3 kg/sq.cm the additional settlement due to wetting was 2 to 4 times as great as at natural moisture conditions. Loess soils with porosity less than 40%, are stable and are largely unaffected by wetting.

Foundation test results reported by Grigorian (5) shows that the curves formed by wires under foundations of different

sizes were identical with the analysis by the elastic theory except the deformations decay more rapidly with depth in the tests. The depth of deformation-zone under strip foundations was found to be larger than under square ones. The lower boundary of the deformation zone in each test is associated with a certain pressure level found to be 0.7 kg/sq.cm for all variants except the smallest, which is almost identical with the initial settlement pressure obtained in the oedometer test.

Pile test results in the loess soils by Finaev (6) shows that the tip resistance is almost constant irrespective of penetration depth. This tip resistance amounts to some 60% of the overall bearing capacity at natural soil moisture, and 65% after wetting. When the tip is in low subsident or non-subsident soils the decrease in bearing capacity under wetting is smaller. The drop in bearing capacity in subsident soil after wetting is due to the decrease in cohesion, from a range of 0.35 to 0.2 kg/sq.cm to a range of 0.1 to 0.05 kg/sq.cm. As the depth of penetration increased the effects of wetting on the bearing capacity decreased; since it was shown that the bearing capacity of pile was basically associated with the lowermost 1 meter section of pile which accounts for over 80% of total bearing capacity. It is thus advisable to drive the piles to depth of about 1 m into a denser low-subsident or non-subsident soil layer.

Stabilization of Soil with Cement

A small percentage of cement may be used as an admixture to soil to change the undesirable characteristics and modify the soil into a more favorable construction material. Increasing the amount of cement to 5 to 15 percent by weight of the soil produces a material, called "soil-cement" which has some of the characteristics of concrete. (16)

Laboratory and field experience during the past few decades has shown conclusively that soil can be hardened or modified adequately by the addition of relative small quantities of cement to produce a strong durable material suitable for low cost paving as well as for others construction purposes.

Test Methods of Soil-Cement

Various tests have been devised to test the strength and durability of soil-cement and cement modified soils. These are summarized in the ASTM Standards (27) and include:

Moisture-Density Test (ASTM 0558-57). - This test is used to determine the optimum moisture content and maximum density either for molding laboratory test specimens or field control.

Wet-Dry Test (ASTM D559-57) and Freeze-Thaw Test (ASTM D560-57). - These tests were designed to determine the affects of the cement with regard to the cementing power in overcoming the stresses set up during cycles of these conditions.

Factors Affecting Soil-Cement

Nature of Soil. - Higher specific surface (surface area per unit volume) in a soil, requires higher cement content

for stabilization. Organic matter, sulfates, and others as well have a great influence on the properties, and usually increase the cement requirement. (16)(18)

Amount of Cement. - Cement content determines the nature of cement-treated soil. The proportion of cement alters the plasticity, volume changes, susceptibility to frost heave, elastic properties, resistance to wet-dry and freeze-thaw cycles and other properties in different degrees for different soil. (17)(18)

Pulverization, Moisture Content, Mixing Compaction and Curing. - Pulverization affects the quality of the soil-cement especially silt and clay soils. The effect of moisture content on the quality of soil-cement largely arises from its influence on the compaction. The efficiency of the mixing and compacting equipment and the time required for mixing and compaction influences both the strength and durability of cement treated soil. The manner in which soil-cement is cured influences the resulting product. As with concrete the strength of soil cement increases with age and like concrete, soil-cement must be kept moist during the initial stages of curing. (18)(19)(20)(21)

Admixture. - Soil admixtures and additives have been used to improve the reaction between the soil and the cement since the earliest projects. Sand, clay, lime, bitumens, emulsion, flyash, and many different kinds of chemical additives, have

been studied and tested for different reactions. Lambe, Moh, and Michaels (7), in a series of reports of laboratory studies, have shown that dramatic improvement in the strength of soil-cement can be obtained by the addition of small amounts of certain chemicals such as $CaCl_2$ and Na_2SO_4 to reduce the amount of cement required and to stabilize some soils which are not responsive to cement alone.

Cement-Modified Soil

Relatively small quantities of cement may also be used as an admixture to change the undesirable characteristics of soils, improve their plasticity or swell characteristic, prevent consolidation and pavement pumping, and increase load-carrying capacity, (16)

DESIGN OF EXPERIMENT

In this laboratory study, a loess soil from the Kansas City, Kansas area was used for the experiments. This raw soil was modified by Type III Portland Cement and sodium sulphate. The effects of the modifications were measured by direct shear, wet and dry test, and unconfined compression tests.

Direct Shear Test for Cement-Treated Loess and Raw Loess Soil

Samples were molded for the direct shear tests as shown in Table 1 to study the effects on the physical characteristics by the cement and sodium sulphate modifications.

Table 1

Experiment Design for Direct Shear Test

Type of Soil	Normal Loads, psi			
	5.1	11.8	22.6	32.8
Raw Soil	1	1	1	1
3% Cement-Treated Loess	1	1	1	1
6% Cement-Treated Loess	1	1	1	1

Resistance to Wet-Dry Test

For the wet-dry test 24 specimens were molded to determine the percentage of wet-dry loss at the standard proctor density as shown in Table 2.

Table 2

Specimens Designed for Wet-Dry Test

Cement Content % by Weight	Sodium Sulfate Content, % by Weight			
	0	0.5	1.0	1.5
0	1	1	1	1
3	1	1	1	1
6	2	2	2	2
9	1	1	1	1
12	1	1	1	1

Another 4 specimens were molded at modified Proctor density, with levels of 0, 0.5, 1.0, and 1.5 percent of sodium-sulfate, all at 6% cement content, to compare the change in wet-dry losses as affected by increased density.

Unconfined Compressive Strength

144 specimens were molded to standard Proctor density as shown in Table 3 to determine the effects of cement and sodium sulfate on the unconfined compressive strength of the soil.

An additional 27 specimens at 6% cement content and 0% sodium sulfate were molded at three different densities as shown in Table 4 to study the relation between unconfined compressive strength and compaction density.

Table 3

Experiment Designed for Unconfined Compressive Strength

Cement Content % by Weight to Raw Soil	Sodium sulfate Content, % of Dry Weight of Soil	Curing Age		
		7 Days	28 Days	90 Days
3	0	3	3	3
	0.5	3	3	3
	1.0	3	3	3
	1.5	3	3	3
6	0	3	3	3
	0.5	3	3	3
	1.0	3	3	3
	1.5	3	3	3
9	0	3	3	3
	0.5	3	3	3
	1.0	3	3	3
	1.5	3	3	3
12	0	3	3	3
	0.5	3	3	3
	1.0	3	3	3
	1.5	3	3	3

Table 4

Specimens Designed for Unconfined Compressive Strength Vs. Compaction Density

Cement Content % by Weight to Raw Soil	Compaction Density	Curing Age		
		7 Days	28 Days	90 Days
6	90% Standard Proctor	3	3	3
6	Standard Proctor	3	3	3
6	Modified Proctor	3	3	3

There were also 24 specimens, molded at two cement levels at three different moisture content as shown in Table 5, to study the moisture content effect to unconfined compressive strength.

Table 5

Specimens Designed for Different Moisture Effect to Unconfined Compressive Strength

Cement Content % by Wt. to Raw Soil	Moisture Content % by Wt. to Raw Soil	Curing Age	
		7 Days	28 Days
6	10	3	3
	20*	3	3
	30*	3	3
12	10	3	3
	20*	3	3
	30*	3	3

*Specimens molded in 2" x 2" x 2" mold.

PROCEDURE AND APPARATUS

Testing on Engineering Properties of Raw Soil and Cement Modified Loess

All apparatus required for these tests listed below was available in the Soil Mechanics Laboratory. This apparatus met necessary ASTM standards for dimension, accuracy or other specified characteristics necessary to assure conformance with test procedures and reproducibility of results.

The following tests were performed to determine the properties of the loess soil used in this study and cement modified loess which was pulverized to pass No 10 sieve after 28 days curing age.

A. Grain size analysis - The pulverized loess-cement was immersed in water for 24 hours before this test was conducted.

B. Atterberg limits and indices test.

C. Direct shear test - Four specimens were molded to standard Proctor density for each level to be tested. Consolidated-quick shearing tests were conducted with four different normal loads as shown in Table 1.

All the procedures of these tests were conducted in accordance with the test procedure recommended by Lambe. (8)

Determining Moisture-Density Relations of Soil-Cement Mixtures

These tests were conducted in accordance to ASTM Designation D558-57, in order to determine the optimum moisture content and maximum dry density of soil-cement with 0, 3, 6, 9 and 12% cement content for a basis of molding wet-dry and

unconfined compressive test specimens. All soil-cement in this research was mixed in the mixer shown in Fig. 1 for two minutes.

Fig. 1. Mixer for preparing Soil-Cement Mixtures

Wet-Dry Test

The specimens as shown in table 2 were molded in accordance with Standard Proctor Compaction Method (ASTM Designation D558-57). The other 4 specimens of 6% cement content level were molded in accordance with Modified Proctor Compaction Method (ASTM Designation D1557-66T).

After specimens were cured in the moisture room for 7 days, they were then subjected to twelve wetting and drying cycles in accordance with the procedures specified in ASTM Designation T135-57, and the percentage of losses was calculated.

Unconfined Compressive Strength Test

The specimens 2.5 in. diameter and 2.5 in. high, were molded in a floating piston type cylinder shown in Fig. 2, under sufficient pressure from the compression machine to provide specimen density equivalent to 90% of Standard Proctor density, Standard Proctor density and Modified Proctor optimum density as shown in Table 4. The specimens were then cured at room temperatures of 20° to 25° C and relative humidity of 100% shown in Fig. 3.

Specimens were subject to 24 hours of complete immersion in water at room temperature prior to testing in unconfined compression test.

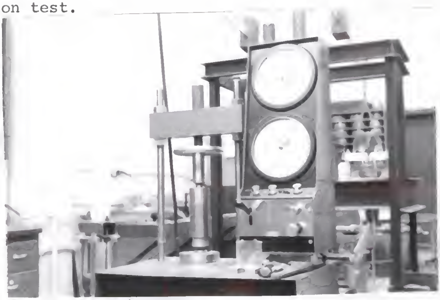


Fig. 2. Compression Machine and Floating Piston Mold.

Fig. 3. Unconfined Compression Specimens Curing in Moisture Room.

PRESENTATION AND INTERPRETATION OF DATA

Testing Results of Engineering Properties of Raw Soil and Pulverized Cement Soil

Cement in smaller amount than that needed to produce soil-cement is used to improve the performance of foundation soils for special purposes.

The results of grain size distribution analysis in Fig. 4 and Table 6, show that the raw soil, consisting predominantly silt size, and relatively high clay content, after treatment by relatively small quantities of cement flocculated, perhaps by a combination of base exchange phenomena and cementing action, to form small conglomerate masses of new soil grains or aggregates of much coarser texture.

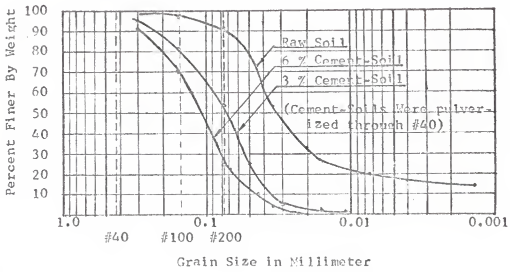


Fig. 4. Grain Size Distribution of Raw Soil and Cement Modified Soil.

Table 6

Raw Soil and Cement-Modified-Soil Grains Size Texture

Soil Type	Grains Size Distribution, %		
	Sand 0.4 - 0.076 mm	Silt 0.076 - 0.005 mm	Clay Below 0.005 mm
Raw Loess Soil	8.5	75	16.5
3% Cement Modified Soil	47	53	0
6% Cement Modified Soil	76	24	0

It is obvious that either 3% or 6% cement modified soil has less clay content and nearly 50% weight of the 3% cement modified soil and 76% of the 6% cement modified soil is sand size.

From the Atterberg limits test results shown in Table 7, and direct shear test results shown in Fig. 5, the cement modified loess soils possess lower plasticity and volume change characteristics than the raw soil and greater load-bearing capacity over a wide range in moisture content. Cement treatment can be used to control shrinkage and expansion of high volume change soil, improve the strength characteristics, and reduce the effect of wet-dry and frost action.

Table 7

Water Consistency of Raw Loess Soil and Cement Modified Loess Soil

Type of Soil	Liquid Limit	Plastic Limit	Plastic Index	Shrinkage Limit
Raw Loess Soil	26.0	23.3	2.6	23
3% Cement Modified Soil	29	NP	-	-
6% Cement Modified Soil	35	NP	-	-

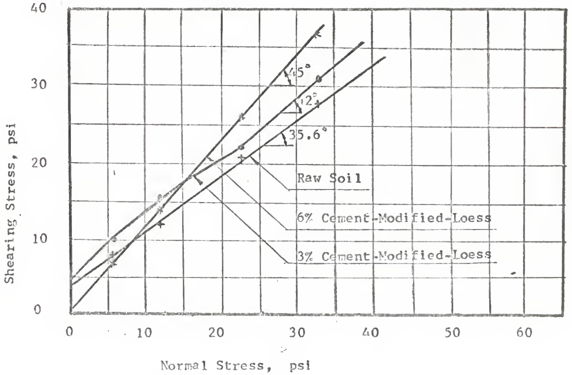


Fig. 5. Shearing Stress and Cohesion of Raw Loess Soil and Cement Modified Loess Soil from Direct Shear Test.

Wet-Dry Test Results

This test was designed to determine the resistance of the soil cement to alternate wet and dry conditions that would produce disastrous results on the raw soil.

Testing results showed that specimens of loess soil treated only by sodium sulfate were completely disintegrated during the first cycles of the test. As seen in Table 8, a 3% cement content level loess-cement did not increase the strength of the soil enough to resist high shrinkage stresses set up during the drying cycle and surface expansion stresses set up during wetting cycles. About 4.5% cement content is required to improve the wet-dry loss to a point within the 10% loss recommended by the P.C.A. test criterion. The loess-cement specimens at cement contents greater than 6% were strong enough to withstand the wetting and drying test with very little loss.

It was also found that the 6% cement content test specimens molded to the modified Proctor density showed less percentage of wet-dry loss as shown in Table 9.

Table 8

Loess-Cement Wet-Dry Test Losses

Cement Content % of Dry Soil	Na_2SO_4 Concentration % of Dry Soil	Weight Losses %
0	0 - 1.5	100
3	0.0 0.5 1.0 1.5	19 20 30 30
6	0.0 0.5 1.0 1.5	0.5 1.0 2.5 4.5
9	0.0 0.5 1.0 1.5	0.0 0.5 1.0 1.5
12	0.0 0.5 1.0 1.5	0.0 0.0 0.5 1.0

Table 9

Comparison of Wet-Dry Test Losses Vs. Compaction Density

Cement Content	Compaction Density	Na_2SO_4 Concentration % of Dry Soil	Weight Losses %
6%	Standard Proctor	0 0.5 1.0 1.5	0.5 1.0 2.5 4.5
	Modified Proctor	0 0.5 1.0 1.5	0 0 1.0 3.5

It is shown in Fig. 6 and Table 8 that sodium sulfate has a detrimental effect on the loess-cement at each of the varying cement content. Fig. 6 indicates that higher sodium sulfate contents causes more cavities to appear in the surface of specimens. The raw soil specimens completely disintegrated in this test and are not included in the photograph.

Fig. 6. Specimens after 12 Cycles Wet-dry Test.

Unconfined Compressive Strength Test Results

Unconfined compressive strength is the most widely used test to determine the strength of cement treated soil. It is considered to indicate the degree of reaction of soil-cement-water reaction or the rate of hardening. For normally

reacting granular soils, compressive strength serves as a criterion for determining minimum cement requirements for construction of soil-cement. The test results for the 7, 28 and 90 day unconfined wet compressive strengths for the loess-cement used in this study are given in Fig. 7 and Table 10.

Lambe, T. W. and Moh, Z. C. (7) reported in 1957 that sodium sulfate is the cheapest agent which could increase the 28 day unconfined compressive strength, 0 - 50% of New Hampshire silt, 50 - 100% for Massachusetts clayey silts, and 20 - 50% for the Vicksburg loess with 1.0% of concentration of sodium sulfate. The test results of this study showed only 0 - 25% increase at higher cement content by sodium sulfate. At sodium sulfate-cement ratios above 25% the strength was decreased by some 5 to 30%, and the surface of test specimens cracked badly.

The compressive strength reduction after 12 cycles of the wet-dry test are shown in Table 11.

The compressive strength of 6% cement content soil-cement at a Proctor Standard compaction density after 12 cycle of wet-dry showed that the strength decreased to only 45 to 65% of the original strength. At higher density and higher cement content the strength of the specimens was not affected by the wet-dry cycle.

The relationship between compaction density and compression strength are shown in Fig. 8. The test results show that the 7 and 28 day compressive strength increased linearly

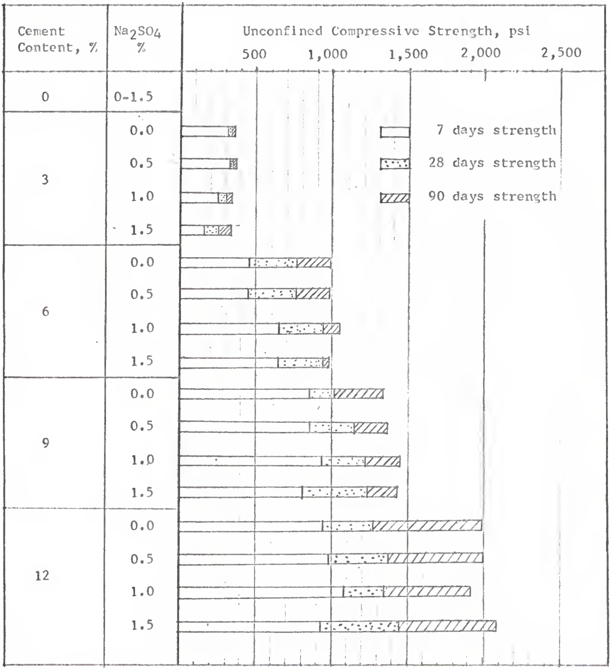


Fig. 7. Compression Strength Vs. Varying Percentage of Sodium Sulfate and Cement Concentration.

TABLE 10

Results of Unconfined Compressive Strength Test.

Cement Content % by Wt.	Na_2So_4 % by Wt.	Curing Age 7 Days No.1	7 Days No.2	7 Days No.3	28 Days No.1	28 Days No.2	28 Days No.3	90 Days No.1	90 Days No.2	90 Days No.3
0	0 - 1.5	0	0	0	0	0	0	0	0	0
3	0	320	336	306	367	356	346	360	365	370
	0.5	320	336	320	336	356	392	367	367	367
	1.0	244	244	244	285	326	300	340	370	370
	1.5	126	143	183	210	265	306	326	358	350
6	0	458	458	500	670	845	765	1,160	915	980
	0.5	484	484	458	715	715	918	1,060	900	980
	1.0	610	680	685	918	960	938	1,080	1,080	1,020
	1.5	640	635	660	980	930	950	940	980	1,040
9	0	835	875	852	1,080	937	1,020	1,300	1,350	1,325
	0.5	815	815	895	1,140	1,090	1,220	1,370	1,370	1,370
	1.0	896	1,120	896	1,220	1,270	1,180	1,570	1,300	1,610
	1.5	815	815	770	1,160	1,300	1,290	1,530	1,430	1,490
12	0	917	977	957	1,300	1,250	1,300	1,940	2,040	1,990
	0.5	1,080	896	1,070	1,360	1,490	1,340	1,940	2,040	2,000
	1.0	1,070	1,100	1,080	1,420	1,395	1,280	1,930	1,830	2,040
	1.5	977	968	875	1,490	1,425	1,550	1,960	2,140	2,280

Table 11

Compressive Strength Reduction After Wet Dry Test.

Cement Content	Sodium Sulfate Concentration	28 - Day Strength (psi)	31 - Day Strength after 12 Cycle Wet-Dry Test (psi)
6	0 0.5 1.0 1.5	760 740 940 910	405 485 485 405
* 6^m modified density	0 0.5 1.0 1.5	980 980 980 980	1,040 985 800 1,000
9	0 0.5 1.0 1.5	1,025 1,150 1,215 1,250	1,000 930 1,000 800
12	0 0.5 1.0 1.5	1,275 1,360 1,360 1,460	1,195 1,235 1,195 1,275

and in proportion to the density of the loess-cement with a rate of 15 psi and 20 psi respectively for each one pound increase in density per cubic feet of loess-cement. For the 90 day strength the rate varied from 40 psi at 90% standard proctor density to 20 psi for the compaction density greater than standard proctor for each pound gain in density per cubic feet.

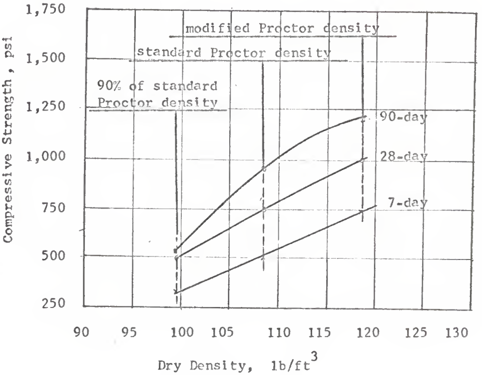


Fig. 8. Dry Density Vs. Compressive Strength of 6% Loess-Cement.

The correlation between the compressive strength and the molding moisture are shown in Fig. 9.

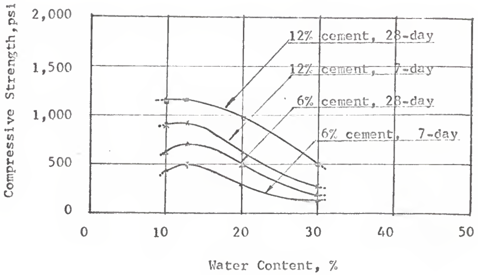


Fig. 9. Moisture Content Vs. Compressive Strength.

Increasing the water content not only decreased greatly the compressive strength but also decreased the workability at water contents above the plastic limit.

Soil-cement was observed as expected to increase in compressive strength with time of curing, with a better than random correlation in both the semi-logarithm and logarithm manner. These relationships can be used to predict the compressive strength of soil-cement at a future date and as an aid in decreasing the cement content of certain soil-cements. It is adaptable for use as a basis to periodically increase the loading on these materials.

The testing results showed a good strength-age linear relationship on the logarithmic plots as shown in Fig. 10 and can be expressed by the formula:

$$\log S = \log A + B \log T$$

S = Strength,

T = Time of curing, and

A, B = Constants

if A = strength of 7 day curing, B = slope of line, then

$$\log S = \log A + B \log \left(\frac{T}{7}\right)$$

The results confirm Circes report of 1962 (9) that A - 4, A - 6 and A - 7 soil-cements produced a good linear relationship on the basis of durability or strength after 7 days of curing. It is recognized that the strength increases with age thus providing a safety factor in the design. Some soil-cement mixtures, develop a strength of over three times (10) the original 7 day strength after additional curing.

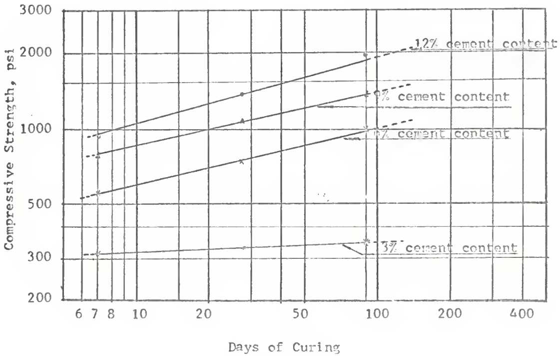
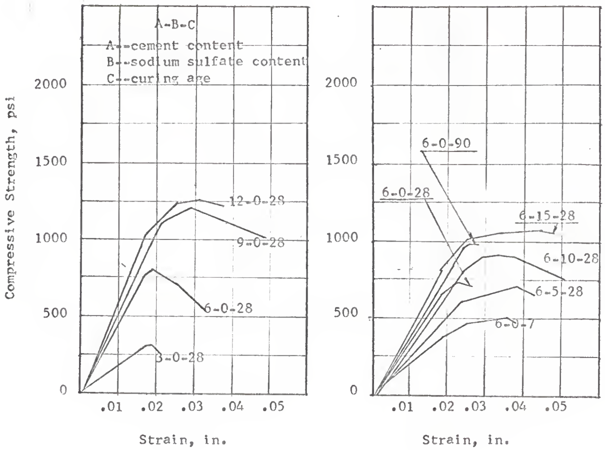


Fig. 10. Curing Age-Unconfined Compressive Strength Relationships.

The modulus of elasticity from the unconfined compression test on the 2.5" diameter, 2.5" height test specimens, showed a very good linear relationship between the stress and strain from 10% to 80% of ultimate strength as shown in Fig. 11.



Elasticity Vs. Cement Content

Elasticity Vs. Na_2SO_4 Content and/or Curing age

Fig. 11. Static Elasticity of Compression.

The static modulus in compression of 28 day curing age computed by the method recommended by the Portland Cement

Association at approximately 33 percent of ultimate strength were 41,000 psi for 3% cement content, 102,000 psi for 6%, 150,000 psi for 9% and 182,000 psi for 12%. These results compared unfavorably to the Portland Cement Association study using 2.8 in. diameter and 5.6 in. height samples. It is quite surprising that the results of this study indicate a modulus much lower than the modulus found in the PCA study.

It was also found that the modulus increases with curing age, with cement content, and with increased sodium sulfate.

CONCLUSIONS

Loess-Cement

The results of this investigation show that beneficial effects are obtained by the addition of cement, even in small amounts to loess soil and lead to the following conclusions:

A. The loess soil used in this study has a natural porosity about 40%, so this soil is a nonsubsident soil. At standard Proctor density, n = 34.5% at modified Proctor density, n = 29%. These compaction densities are quite satisfactory for stabilization of foundation soils affected by wetting.

B. 3% loess-cement was not strong enough to resist wet-dry test loss by P.C.A. criteria. The wet-dry loss decreased with increased cement content and/or by increased density.

C. Sodium sulfate had an unfavorable effect on wet-dry resistance, a 1.5% sodium sulfate had a delerterious effect to low cement content and early curing age strength, and a very slight effect on long term strength contrary to existing information from the literature.

D. Loess-cement strength increased rapidly as cement content increased.

E. The compressive strength decreased with the number of wet-dry test cycles, the lower cement content loess-cement decreased at a higher rate than the higher cement content level.

F. At the same cement content level the higher density showed a lower loss of compressive strength during the cycle

of the wet-dry test.

G. Higher molding moisture not only decreased the strength but also caused poor workability.

H. At all levels of cement content, the loess-cement mixtures showed that the strength had a good linear logarithmic relation with the age of curing.

I. Loess-cement had a static modulus (E) in compression from $0.04 \times (10)^6$ psi to $0.18 \times (10)^6$ psi. This modulus increased with increasing curing age and cement content.

J. Cement treating increased the size of soil particle by conglomerating the fine particles and leads to large particles of cemented-loess which have no plasticity and small shrinkage characteristics.

K. Cement-modified loess has a larger friction angle than the raw soils even at lower cement contents such as 3%.

L. Loess soil under a moisture of 10% were easy to pulverize for the purpose of cement stabilization.

Application of Loess-Cement to the Spread Footings in the Loess Mantled Regions.

Critical Conditions of Loess Behavior on Foundations.

A. High porosity is the most detrimental parameter on the subsidence of loess soil. Non-subsident varieties are usually of low-porosity (less than 40%).

B. The subsidence capacity of loess is mainly reflected by vertical settling due to reduction in the void ratio on wetting.

C. In view of strain and pressure relationships the loess soil acquired the bearing capacity within the compression zone to a depth of approximately 1.5 B (B = width of foundation).

D. It was established that within the compression zone subsidence occurs under a pressure of 2 kg/sq.cm and in part under 1.5 kg/sq.cm whereas at 1 kg/sq.cm no subsidence will occur. Loess soil wetted under a low (1.0 - 1.25 kg/sq.cm) pressure does not subside significantly while at 2 - 3 kg/sq.cm the additional settlement due to wetting is 2 to 4 times as much as under natural moisture condition.

E. The deformation zone depth of the strip variants was found to be larger than square ones and this is also in accordance to elastic theory.

F. The lower boundary of the deformation zone was found to be associated with a pressure level of 0.7 kg/sq.cm. almost identical with the initial settlement pressure obtained in oedometer tests.

G. The deformational properties at saturation for the high porosity loess in triaxial tests showed stress-deformation anisotropy. In this reference a solution to the problem of deformation prediction is obtainable by means of field studies on test foundations of different shapes and size.

H. The drop in bearing capacity in subsident soil on wetting is due to the decrease in cohesion from 0.35 - 0.2 kg/sq.cm to 0.1 - 0.05 kg/sq.cm. Pile test results showed that as the depth of penetration increased the effect of wetting on the bearing capacity decreased.

Criteria for Applying Loess-Cement Stabilization to Spread Footings.

A. Increase the footing capacity by increasing the width of the loess-cement block to enable spreading the structural load to a contact pressure (p_o') to under 1.25 T/ft^2.

B. Increase the foundation depth by using loess-cement to form a block to a depth less affected by the water content.

C. The loess-cement foundation block should be large enough to encompass the 0.2 p' equipressure line, or at least large enough to distribute the loading to $p_o' \leq 1.25$ T/ft^2 as shown in Fig. 12.

D. Check the loess-cement block by shearing stress, using 7 day strength as control criteria.

E. Check the soil bearing capacity under soil-cement block by the plastic equilibrium theory equation as proposed by Karl Terzaghi (11).

F. Because of the high bearing capacity of loess-cement the size of structural footing can be decreased in size by some amount in accordance with the size and the strength of loess-cement block.

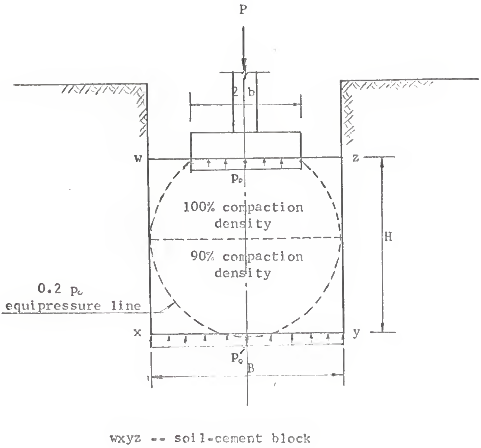


Fig. 12. Soil-Cement Block Under Spread Footing.

RECOMMENDATIONS OF FURTHER RESEARCH

For the ever increasing structure activity in the loess mantled areas, the stabilization of loess soil by cement for spread footings of structure is both practical and theoretically sound for field practice. It is urgent that further research be inaugurated to develop economical machinery and procedures for the purpose of mixing and compaction of the loess to form the foundation soil-cement block.

APPENDIX

Review of Additional Literature Pertaining to the Problem of Stabilizing Loess for Foundation

Physical Properties of Loess Soil

Soil grain properties

The most unique feature of loess is the grain-size distribution. Silt particles (0.05 - 0.005 mm. by ASTM classification) are predominant, with the percentage reaching 70 - 90%. Loess soils are classified as loams or sandy silts. Natural loess soils are characterized by lack of stratification and foreign inclusion. The reworked derivations, however, may be stratified and contain rock debris.

The wind blown silt deposits of the United States have many similar engineering characteristics. They are generally uniform in texture, light brown, and somewhat cohesive. They consist of 50 - 90% of silt-size particles and referred to frequently as rock-flour silt. (3)

Loess Soil structure and water-soil consistency

The mineral composition of silt and sand fractions, consist of quartz, feldspar, carbonates and, occasionally, mica. The clay particles consist of colloidal material. (1)

The moisture content is extremely variable and ranges from a low of 3-4% to full saturation of some 30% below groundwater level. In soils in the weathering zone, low moisture contents (7 - 10%) are most common. The moisture content depends on climatic conditions, on the thickness of the loess layer, and on the depth to impermeable strata. In arid regions, where loess layers are of considerable thickness

and underlain by sands, the moisture content is usually not widely variable (8 - 12%), but without any special regularity. When underlain by clays, the moisture content increases systematically with depth.

The bulk density of loess soils under natural moisture conditions fluctuates between 1.43 and 1.58 g/cm^3. The specific gravity is rarely less than 2.64 or more than 2.72. Porosity is the most characteristic parameter of subsident loess soils and varies from 40 to 62%. The most common range being from 44 to 53%. Non-subsident varieties are usually of low porosity, less than 40%. (1)

Leonards (2) in his book, Foundation Engineering, notes that the liquid limit of wind-blown silt is approximately 30, and the plasticity index is approximately 6. Wind-blown silts, in their natural states, are well drained because true loess deposits have characteristically a very high vertical permeability. This peculiarity in permeability permits vertical percolation of water with little or no horizontal movement. If loessial soils are dry, they are difficult if not impossible to compact, if excessively wet, loess exhibits quick properties in either cut or fill sections. Because of the porous structure, a large "shrinkage factor" is required in estimating earthwork. Frost heave is common in deep silt deposits with high ground water table in cold humid climates.

For foundations to be located on silts the in-place density, natural moisture content and strength and consolida-

tion characteristics of the silt must be determined under conditions of complete saturation. The behavior of these materials is very sensitive to small changes in density and moisture content, making them "tricky" to handle in both foundation and embankment work. (3)

Shearing resistance of loess

The relation between normal pressure and shearing resistance of silt and silty sand is similar to those for clean sand. The values of ϕ_s, obtained from the slow shear test, range from about 27° to 30° for the loose state and 30° to 35° for the dense state. (3) These values are almost as great as those for sand.

The relation between water content and unconfined compressive strength for a silt or loess is shown as Figure 13. (11) As the water content approaches the shrinkage limit, the strength increases. At the shrinkage limit, air invades the voids of the specimen and the strength decreases gradually until the degree of saturation becomes approximately 10%. Thereafter it increases and becomes greater than at the shrinkage limit.

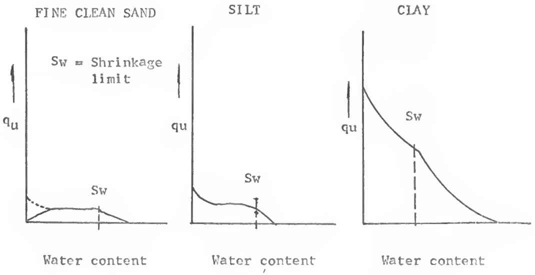


Fig. 13. q_u vs. water content for clay, silt and fine clean sand.

The characteristics of a loess deposit are shown in Fig. 14 and Table 12. (12)

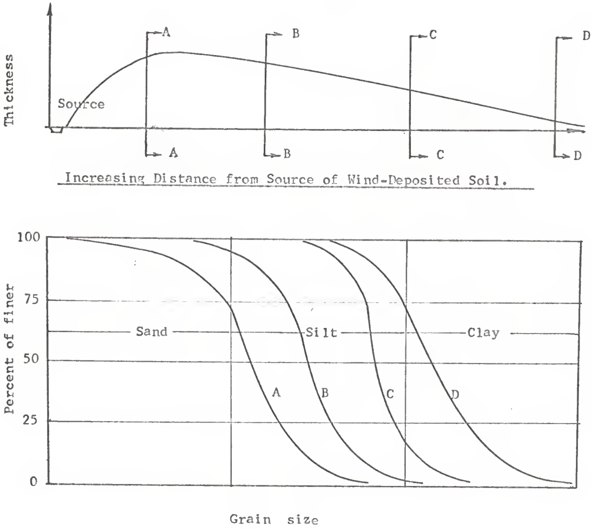


Fig. 14. General Thickness and Grain Size of Loess Deposits.

Table 12

Deposit of Loess Soil and its General Characteristics.

Region	Thickness	Cohesion	Ø	Min. water table
A	100 - 200'	0	25 - 30°	60 - 80'
B	60'	0	15 - 20°	20 - 30'
C	30'	2 - 5 psi	10 - 15°	0'
D	10'	5 - 10 psi	5 - 10°	0'

The allowable bearing value for loess under different moisture conditions shown in Table 13 are based on Terzaghi's finding, as illustrated and defined on page 83 and 84.

Table 13

Typical Allowable Bearing Values for Loess Soil at Different Moisture Content Ranges from Four Deposit Regions

Deposit region	Moisture Content		
	Dry	Moist	Saturated
A	2500	3000	2000
B	2000	2500	1500
C	1500	2000	500
D	3000	1500	1000

From the above the allowable bearing value is obviously greatly reduced by saturation independently of the calcarious cement. This calcarious cement is relatively insoluble except in the long term. The basic cause for the reduction in strength is apparent cohesion. This apparent cohesion results from the mechanism shown in Fig. 15.

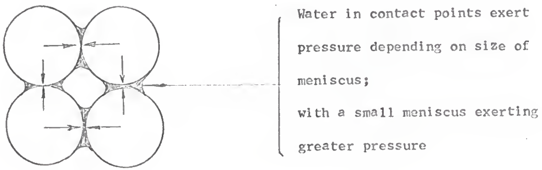


Fig. 15. Water surface tension effects on shearing resistance.

The "strength" of deep loess deposits is dependent upon intergranular friction and is thus greatly affected by apparent cohesion. This is because the water table is usually low so that much of the deposit never becomes saturated. Down wind the soil is thinner and the strength is more and more dependent upon the cohesion characteristics. In critical areas the water table may be at or near ground level thus destroying the apparent cohesion and causing a decrease in intergranular friction. Soils in such areas would exhibit low strength and low bearing capacity. Still farther down wind the clay content

becomes high enough that cohesion is appreciable and this cohesion is not destroyed by the saturation caused by a high water table.

Illustrating the above theory the strength downwind would follow the curve shown in Fig. 16.

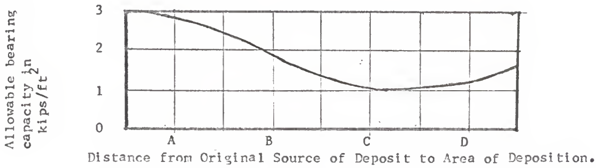


Fig. 16. Relationship Between Distance from Original Source of Silt and the Area of Deposition Vs. Allowable Bearing Capacity.

It is relatively safe in areas A and B to design spread footings if the site were properly protected from wetting, but further downwind, as the loess becomes thinner and the water table approaches the surface (even though the material is very similar) many foundations would fail if based on the same allowable bearing capacity as used in areas A and B.

It was found that structures on loess 100' thick, as in the Council Bluffs, Iowa area, with contact pressures in the range of 2000 - 3000 PSF rarely rail while in loess 10' deep structures exerting no more than 1000 psf commonly fail. (12)

Foundations on Loess Soil

Significant characteristics of loess for foundation design

Loess with a plasticity index near zero, has characteristics similar to a fine sand. The characteristics of true loess deposits are extremely different from those of waterlaid silts. The calcarious or clayey binder present in most true loess deposits causes appreciable cohesion in spite of its relatively high void ratio. Experience indicates that such deposits may be capable of sustaining loads of several kips per square foot without appreciable settlement. The quality of the binder is likely to differ from point to point, partly because of erratic variations in the water content, thus the strength of the deposit is likely to vary widely within short distances. Thick deposits of true loess are usually unsaturated but, if they become saturated, some of the binder is likely to dissolve or soften, and cause the deposits to lose cohesion. In this event the structure of the soil collapses and the void ratio decreases significantly. This is likely to cause settlement of the ground surface sufficient to damage buildings.

Denisov, (13) reported that the subsidence capacity of loess is mainly reflected in vertical settling due to compaction on wetting. This vertical settling signifies transition from an underconsolidated state to one of normal consolidation. Soil strength is known to be directly dependent on soil density. However, underconsolidated soils exhibit considerable strength

in spite of low density or of porosity of 45 percent or higher. This is attributable to the formation of the structural bond preceding the adjustment of porosity to pressure. On wetting, under natural pressure, rapid disruption of the structural bond sets in and the particles assume a more compact array. This process, due to the wedge effect of thin water films, continues until the soil has achieved normal consolidation.

The following types of loess soil are distinguished according to their reaction to wetting: (1)

A. Those subsiding under natural soil pressure (subsidence in the proper sense);

B. Those subsiding only under additional pressure applied by the weight of a structure (additional settlement); and,

C. Those loess soils with stable structure, indifferent to wetting, and $n \leq 40\%$.

Field experiments on wetting of loess soils (1) revealed an important property regarding their subsidence deformation which may reach 1 to 2 meter. It was determined that there were two zones in thick loess layers. An upper layer where the soil has lost its capacity for subsidence through the action of atmospheric wetting under natural pressure; and, a lower one where this capacity is still intact.

From the above it must be understood that loess soils possess high porosity; are extremely sensitive to moisture change; and, that increased moisture content will cause large

settlements that will be detrimental to all types of structures which must be located on them.

Nature of Subsidence of Loess Soil

In the thin deposits of loess soil, subsidence of a soil is estimated in terms of the strain (d_s) that occurs in consolidation tests on undisturbed samples. The measurement of strain is expressed in the following equation: (1)

$$d_s = (h_1 - h_o) / h_o$$

where h_1 = height of soil specimen at natural water content under a real pressure (P_r),

h_2 = height of specimen after wetting under the same pressure,

h_o = height of specimen at natural moisture content under natural pressure (P_{nat}).

Empirically a soil is classified as subsident if $d_s \geq 0.01$, and non-subsident if $d_s < 0.01$.

It can be shown (1) that a thin loess strata, which is non-subsident under natural pressure, will show subsidence when subjected to additional load as in the case of a foundation, also, the subsidence will normally occur within a depth of 1.5 B, where B is the width of the foundation.

Data on the dependence between strain and pressure determined in oedometer tests were presented by Drannikov, (1) in experiments on Ukraine loess. It was established that within

the compression zone subsidence occurred to some extent under a pressure of 1.5 kg/sq.cm, and completely at 2.0 kg/sq.cm, whereas at 1 kg/sq.cm no subsidence occurred. Since oedometer tests do not provide a complete pattern of the subsidence process, they were supplemented by a series of field tests of static loading using settlement plates 5,000 sq.cm in size. After stabilization of settlement under a load, the soil was wetted and additional settlement recorded. The results are presented in Table 14.

Table 14

Subsidence of Settlement Plate on Loess Soil
Due to Loading and Wetting

Test Site	Load, kg/sq.cm	Settlement, unit mm		
		At Natural Moisture Content	Additional After Wetting	Total
No. 1	1.0	8.10	3.55	11.65
	1.5	8.30	6.30	14.65
	2.0	11.55	11.38	22.93
No. 2	0.5	3.8	1.7	5.5
	1.0	4.9	1.4	6.3
	2.0	7.4	14.7	22.1
	3.0	10.7	41.7	54.4

The above data confirmed that soil wetted under a low (1.0 - 1.25 kg/sq.cm) pressure does not subside significantly, while at 2 - 3 kg/sq.cm the additional settlement due to wetting is 2 to 4 times as much as under natural moisture conditions.

Grigorian undertook a study of deformationsof loess soil under building and structure foundations. (5) His study of deformational properties of saturated high-porosity loess in triaxial tests showed stress-deformation anisotropy with the greatest strain recorded horizontally even under uniform all-round pressure. In these circumstances the loess specimen cannot be considered as elementary. A solution to the problem of deformation prediction is obtainable by means of field studies on test foundations of different shapes and sizes. Results of such a study, carried out in Dushanbe, found that the linear-deformation theory is admissible for the stress distribution in loess at normal structural loads not exceeding 3 kg/sq.cm.

As with any soil, deformation of loess may be predicted in two ways: (A) on the basis of the deformation of a volume element; and, (B) on the basis of loading tests on model foundations. The stress distribution in the soil under load is assumed, in both cases, according to the linear-deformation-elastic theory. Engineering practice is mostly concerned with the linear vertical deformation of loess, which is due to volume changes in each volume element of the soil, subjected

to σ_1, the vertical principal stress and σ_2 other principal stress. The deformation properties of loess specimens at different states of stress were studied in the triaxial test.

A series of tests by Grigorian on 12 cm x 6 cm diameter specimens under uniform pressure ($\sigma_1 = \sigma_2$) yielded uniform vertical and lateral deformation. Undisturbed Chir-Yurt loess samples, with an initial porosity of 46%, were subjected to 3 kg/sq.cm all-round pressure at 7 - 10% natural moisture content and at full saturation. Lateral strains e_2 exceeded their vertical counterpart e_1 in all tests. By the elastic theory these vertical and lateral strains must be equal irrespective of specimen size. The possibility of other factors (anisotropy, mode of saturation, etc.) was investigated and rejected. The discrepancies between vertical and lateral deformation were found to be due to specimen shape and dimensions.

In 1962 triaxial tests were carried out by Grigorian on saturated Nikopol (Ukraine) loess specimen 9 cm in diameter (d), with heights (h) of 9 cm and 18 cm. The initial porosity was 46%, and all round pressures of 1 and 2 kg/sq.cm were used. At $h/d = 2$, e_2 exceeded e_1 by a factor of 5 to 17, whereas at $h/d = 1$ the ratio e_2/e_1 was 0.9 - 1.2. In these circumstances, the elastic theory was inapplicable to loess specimens with high initial porosity. It is known that the relationship between volume strain and σ_1 in saturated loess specimens is non-linear. The residual compressive deformation on unloading exceeds 90% of the total deformation. It follows

that the saturated loess is a plastic body, in which the stress-deformation relationship depends on the shape and dimension of the specimen as well as on the properties of the material in the specimen. (5) In view of this anisotropy, saturated loess specimens, with high initial porosity, cannot be considered as elementary in triaxial testing irrespective of size.

For loess specimens with natural (low) moisture content and high initial porosity, the vertical to lateral deformation ratio under uniform all-round pressure of 1 - 3 kg/sq.cm did not exceed 2 at h/d = 2. These specimens are known to have a stable structure. For dry loess, the compressive deformation vs. pressure is almost linear in the low pressure range. However, it is more concerned with subsidence predictions for saturated rather than for dry loess, since dangerous deformations are invariably associated with wetting. A field investigation was undertaken accordingly on typical central Asian (Dushanbe) loess soil by Grigorian. The deformation zone, and the deformation pattern of individual horizontal layers within it, were studied. The physical characteristics of the test site soil were as follows: Natural moisture content, 10 to 13%; dry density, 1.29 to 1.35 g/cm^3; specific gravity, 2.69; porosity, 49 to 52%; liquid limit, 32; plastic limit, 19; plasticity index, 13.

Granulometric Composition

Size, mm	% Smaller
0.05	94.8%
0.005	19.8%

This indicates that 5.2% of the material is sand size, 75% silt size, and 19.8% clay size.

The site consisted of homogeneous subsident soil. (5) Four test foundations were laid; two square $0.6^m \times 0.6^m$ and $1.2^m \times 1.2^m$ and two strip-shaped $0.6^m \times 4.0^m$. Strain determined in oedometer tests on soil specimens drawn from the deformation zone under the foundations, ranged from 8 to 10%. The foundations were sunk 15^{cm} below ground level. Marks for determining the boundary of the deformation, and deformation pattern within it were established and an automatic electric loading system ensured uniform settlement.

Load was effected at 0.5 kg/sq.cm intervals to a maximum of 2.0 kg/sq.cm, with each level maintained until deformation was stabilized. On stabilization at the final level, the soil under the foundation was wetted. Surface settlement due to the wetting followed. Wetting was applied throughout the depth of the deformation zone, and discontinued on stabilization with settlement not exceeding 2^{mm}. After unloading, a pit was dug through the foundation, exposing the marks. Representative soil samples were taken for density and water content determination, and the deflections of wires between the marks recorded (as the Fig. 6). Most of the deformation occurred during wetting. Settlements were calculated by formula:

$$S = \sum_{1}^{n} \sigma_i H_i m$$

σ_i = strain determined for each layer in oedometer tests,

H_i = thickness of layer (cm),

n = number of layers,

m = coefficient of working conditions, taken as 2 down to 1.5 times the foundation width and 1 below that depth.

Table 15

Calculated and Actual Settlement Below Test Foundation. (5)

Foundation No	Dimensions (m)	Settlement (cm)	
		Calculated	Actual
1	0.6 x 0.6	15.0	30.0
2	1.2 x 1.2	28.0	24.0
3	0.6 x 4.0	25.0	29.0
4	0.6 x 4.0	25.0	25.0

Table 15 shows a comparison between calculated and actual settlements for all variants except the smallest. A settlement diagram was constructed with the aid of the wire deflections for the midpoint of each foundation as shown in Fig. 17 and 18. The total vertical stresses due to soil weight and external load were plotted for comparing the depth distribution of the soil deformation and stress distribution along the foundation center line. The stresses due to the external load were deter-

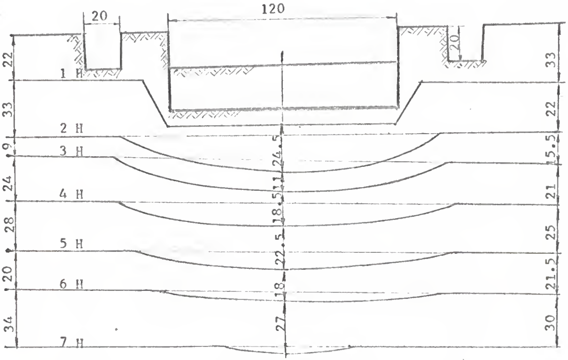


Fig. 17. Deflections of wires 1.2 x 1.2 m variant. (5)

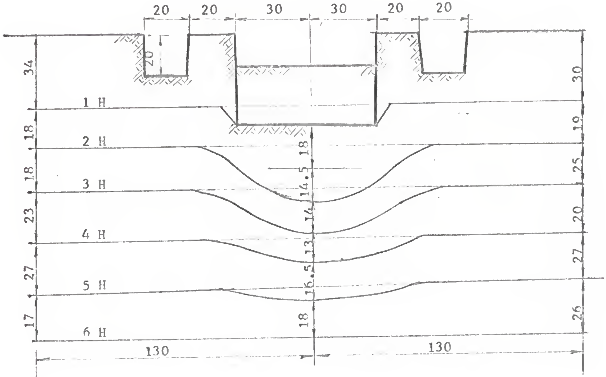


Fig. 18. Deflections of wires 0.6 x 4.0 m variant. (5)

mined by elastic theory Fig. 19. The analogy between the curves is evident, except that the deformations decay more rapidly with depth in the tests than the theoretical computation would indicate.

The ratio of deformation-zone depth to foundation width (h/b) for the 0.6 x 0.6 m variant is 2.05; for the 1.2 x 1.2 m variant, 1.38, and for two 0.6 x 4.0 m variant, 2.8. The deformation-zone depth for the strip variants was found to be larger than square ones, also in accordance with the elastic theory. The lower boundary of the deformation zone in each test is associated with a certain pressure level found to be 0.7 kg/sq.cm for all variants except the smallest, which is almost identical with the initial settlement pressure obtained in oedometer tests.

As initial collapse pressure, Grigorian proposed the level corresponding to a strain of 0.02 obtained from the pressure-deformation relationship in the oedometer test. (5) In Fig. 19 the upper limits of stress curves correspond to the base pressure and the lower limit to the initial settlement pressure (0.7 kg/sq.cm). The linear deformation theory may be used satisfactorily for determining the stress distribution.

Strainswere determined for the center of each layer bounded by the horizontal wires, under the square variant, upper layers to a depth corresponding to h/b = 0.2, were found to undergo insignificant compaction unlike the strip variant, this was also the case in tests with small circular foundation models on sand (Eggestad, 1963). Apparently, tangential

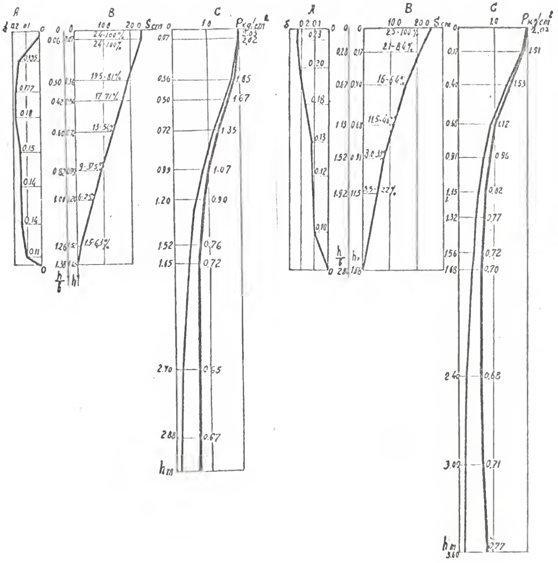


Fig. 19. Axial Stress and Deformation Curves; Left, 1.2 x 1.2 m Variant, Right, 0.6 x 4.0 m Variant. A, Compressive Strains; B, Compressive Deformations; C, Stress Curves in the Soil (light line = external load; bold = total load).

stresses play an important role in the deformation process. The zones of largest compressive deformation coincide with those of highest tangential stresses.

Footings and Raft on Loess Soils

Because soft loess is not capable of supporting footing foundation, structures on such deposits must be founded on piles, piers or rafts established at such level that the weight of the excavated material is approximately equal to that of the building.

The values of N of loess deposit usually vary from 4 to 20 (3) but the standard penetration test does not appear to be a sufficiently reliable indication of characteristics of loess to serve as a basis for design because the uniform consistency and high void ratio of loess facilitates penetration of the sample spoon, but it still proves useful in establishing the uniformity or nonuniformity of a deposit. Performing load tests for determination of allowable soil pressure is advisable. If the material appears to be uniform, three or four load tests may be sufficient for an average sized structure. Otherwise, several tests should be made at locations where the number of blows is a maximum, at a minimum as well as where average conditions prevail. The load test should be made in pit not smaller than 4 by 4 ft in plan and the bearing plate should be 1 ft square. No surcharge should be placed around the loaded area. The load should be increased until failure occurs or until the pressure beneath the test plate is at

least three times that contemplated beneath the footings or raft. A load-settlement curve should be plotted on the basis of the results of each test. If the load settlement curve exhibits a point at which the settlement begins to increase rapidly for small increment of load, the allowable soil pressure should not exceed one third of the load corresponding to this point. (3) Moreover, in any event, the allowable soil pressure should not exceed the value at which the settlement of test plate is equal to 1/2 in.

Upon completion of a footing or raft foundation on loess, special caution must be taken to ensure the water table will remain well below footing.

Piers and Piles in Loess Soil

Piers are identified as a concrete footing which depth is greater than the width and is constructed as a continuous beam under the structure. Piling as defined as a long slender column drilled or driven into the soil to a firm material below the soft stratum and separated from the structure by a member commonly called the pile cap or beam. Piling can be subdivided into two groups:

A. Point bearing pile which transmits the structural load to a hard layer below the point of the pile.

B. Friction pile which develop load carrying capacity by friction between the side of the pile and the surrounding soil.

In loess when the supporting capacity is inadequate the loads are transmitted by piers or piles through the loess to underlying firm material. Excavation for piers can usually be carried out with ease because the material will stand in vertical slope with little lateral support, it is essential to prevent the accumulation of surface water in the bottom of the excavation because such water may cause a reduction of the cohesive bond and thus lead to construction difficulties due to sloughing and caving of the excavation.

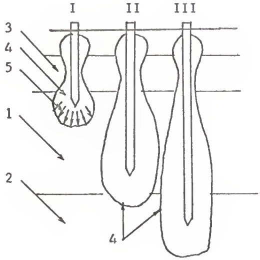
Structures may be established on piles of point bearing or frictional type. If the deposit is relatively thin, the piles may pass through the loess and into an underlying stratum where firm support may be obtained. If the deposit is thick the piles may behave as friction units and the bearing capacity of individual friction pile in loess can be determined reliably only by load test. Tapered piles are usually advantageous for lateral deformation is greater than vertical.

In 1967 Finaev (6) presented the results of pile-resistance test in subsident soil. The tests were carried out at two sites with approximately equal stratification, subsident loess loams, underlain by practically subsident-free sandy loam at 5 - 8 m. The bearing capacity of piles, both penetrating, for the subsident layer was determined for three cases in Fig. 20.

A. tip of pile in subsident soil with a subsident layer under the compacting zone 4.

B. tip in subsident soil, with the compaction zone merging non-subsident soil.

C. pile penetrating all the way through the subsident soil.



(1) subsident loess.

(2) non-subsident loess.

(3) zone of maximum subsident.

(4) zone of compaction.

(5) pressure transmitted from pile load and overburden at the base of the compaction cone.

Fig. 20. Scheme of Pile Action.

The test results showed that the bearing capacity of a pile, completely embedded in subsident soil is considerably reduced by wetting. The soil resistance of metallic pile driven to a depth of 3.25 m in the subsident zone decreased by 46%, tip resistance by 38% and skin friction by 56%. After wetting the soil when the tip is in low-subsident or non-subsident soil, the decrease in bearing capacity is smaller. For example a pile set at a depth of 6 m (zone 4) showed a decrease in resistance under critical load of about 25%, tip resistance by 18% and skin friction by 36%. Under ultimate

load (design loadings that result in settlement not exceeding 10 mm) the relative decrease in resistance on wetting is slightly smaller. The drop in bearing capacity in subsident soil on wetting is due to the decrease in cohesion, from a range of 0.35 - 0.2 kg/sq.cm. to a range of 0.1 - 0.5 kg/sq. cm. The pertinent test results under static load are summarized as:

A. As the depth of penetration increases the effects of wetting on the bearing capacity decreases. Fig. 21. This is attributed to the decrease of the subsident capacity with depth.

B. Test results also show that tip resistance is almost constant irrespective of penetration depth, namely about 60% of the overall bearing capacity at natural soil moisture, and in the order of 65% on wetting. Thus pile design in loess soils should preferably be based on increased penetration depth, and always a minimum of all the way through the subsident layer, so wetting will have a negligible effect on the bearing capacity.

Pile tests under wetted conditions are a serious problem for the building industry since they are difficult to carry out directly on most construction sites. In view of this, it is proposed by Shishkanov, 1962 and Dezhin, 1964, that design resistance of pile load (L) in loess soil under wetting conditions be determined according to the bearing capacity at natural content:

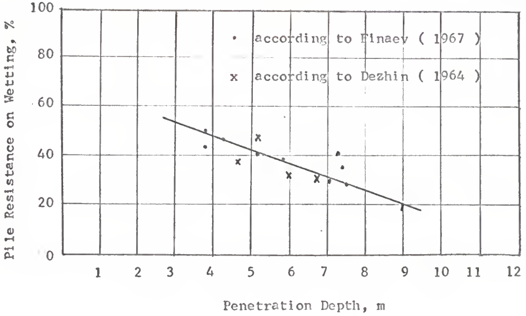


Fig. 21. Reduction in Bearing Capacity on Wetting Vs. Penetration Depth.

$$L = m \cdot L_{ult \cdot n}$$

m = coefficient of working conditions, allowing for the effect of wetting on the bearing capacity.

$L_{ult \cdot n}$ = ultimate resistance of the pile at natural moisture content.

The coefficient m depends on the medium surrounding the pile tip (subsident or nonsubsident). For a medium with 3% settlement and more, m = 0.4 to 0.5; for 1 to 3%, m = 0.6; for complete penetration of the subsident layer with the tip in soil with relative settlement below 1%, m = 0.7.

It is also found that average specific friction in the top 3 m zone of the pile, (where soil has e = 0.75 to 0.95,

strain at 2 kg/sq.cm from 3% to 10%) were 2.76 to 3.88 T/sq.m at natural moisture, and 1.37 to 2.88 T/sq.m under wetting condition. It was also shown that the bearing capacity of pile basically associated with the lowermost 1 meter section of pile, which reacting with the adjoining soil, account for over 80% of total bearing capacity. It is thus advisable to drive the piles to depth of about 1 m into a denser low-subsident or non-subsident layer.

Some Other Treatments to the Foundation in Loess Soil

Vast area of the world, are underlain by loess deposits up to 50 m in thickness. In order to develop this region, designers have to solve these foundation problems. Thermal and chemical methods such as silicate treatment and plastic and slag-grouting have been successfully applied in many cases, yet they are expensive and elaborate and cannot be recommended except in emergency situations.

In 1967, Lehr, (14) based on 6 years' laboratory and field work, presented a new suggestion for preventing structural failure due to subsidence by "earth piles". Loess soil becomes insensitive to wetting below a level of n = 40% and this safe state can be achieved by reducing the natural porosity by compacting to depths of 15 m which is feasible forming the "earth piles". A reliable method, both simple in practice and theoretically logical has been well developed. A cylindrical tube some 45 cm in diameter of suitable shape and dimensions is driven into the earth to the desired depth

by means of percussion drilling pressing the content of the hole laterally into the surrounding earth and thus compacting the soil.

Stabilization of Soil with Cement

General Description of Cement Stabilization

A small percentage of cement may be used as an admixture to soil to change the undesirable characteristics and "modify" the soil into a more favorable construction material. And increased the amount of cement to 5 to 15 percent by weight to of soil to produces a material, called "Soil-cement" which is stronger and more durable than the untreated soil.

Soil cement was first employed in road construction in the U.S.A. in 1933, and since then has been used on an increased scale until by 1948 there were many millions of square yards of this type of construction in all parts of the world. The economy and durability of engineered soil-cement streets and highways led in the 1940's to successful experiments in the use of soil-cement for lining canals and reservoirs protecting levees, and proved a successful application for the slope protection on earth dams.

During the construction of the Bonny Reservoir, (15) two separate soil cement test sections were constructed. One used fine silty sand and required 12 percent cement: the other used medium silty sand requiring 10 percent cement. The test site was selected for conditions of extreme exposure

to freezing and thawing, wetting and drying, and wave action. Damage to both of the soil-cement sections has been negligible. Ten years after construction, cores drilled from the soil-cement facing showed average compressive strength of 2000 psi and 2160 psi, respectively for the fine silty sand and medium silty sand soil. The 28 day strength of the medium silty sand was about 1200 psi for 10% cement content.

Laboratory and field experience during past decades has shown conclusively that soil can be hardened or modified adequately by the addition of relatively small quantities of portland cement to produce a strong durable material suitable for low cost paving as well as other above mentioned purposes.

ASTM-AASHO Test Methods for Soil-Cement

Soil-cement mixtures are controlled by several ASTM and AASHO standard test methods. The most important of these are:

Moisture-Density Test (ASTM D558-57, AASHO T134-57).- The moisture-density test is used to determine the optimum moisture content and maximum density, for molding laboratory test specimens, and also in the field during construction to determine the quantity of water to be added and the density to which the mixture is to be compacted.

Wet-Dry Test (ASTM D559-57, AASHO T135-57). - The wet-dry test was originally designed to determine whether the hardened soil-cement would soften from exposure to severe moisture

variations and alternations of wetting and drying, that have produced disastrous results in soil without cement stabilization.

This test is used to determine the loss of material from formed specimens of soil cement after 12 cycles of wetting and drying as described in the standard test methods.

The wet-dry test is particularly severe on soil-cement specimens molded from soils that contain relatively high percent of silt and clay. The drying cycle sets up high shrinkage stresses in the specimens, these stresses are released when the wetting cycle start, and high surface expansion stresses are set up. The wet-dry test is also rather severe on the poorly graded sand soil-cement specimen, but greater soil-cement losses are permitted for satisfactorily hardened sandy soil-cement mixtures. (16)

Freeze-Thaw Test (ASTM D560-57, AASHO T136-57). - Alternate freezing and thawing is used to show whether the cement is reacting favorably and how effective the cementing power is in overcoming the expansion of water in the voids of the soil-cement as the water freezes. In the loam, and silty clay loam soil-cement mixtures, the test shows:

A. Whether there is sufficient cement in the specimens to overcome the expansion of the water freezing in the void.

B. Whether there is sufficient cement in the specimens to overcome the formation of ice layers. (16)

Factors Affecting Soil-Cement

The factors affecting soil-cement mixtures are described as follows:

Nature of Soil. - Higher specific surface (surface area per unit volume) in a soil, requires higher cement content for stabilization. Organic matter, sulfates, and other constituents as well have a great influence on the properties of cement-treated soil, and usually increase the cement requirement.

As a conclusion table 16 shows the amount of cement required to stabilize different groups of soil to meet the standards set for the wet-dry, and freeze-thaw tests.

Type and Amount of Cement. - For a given soil that react normally with cement, the cement content determines the nature of cement-treated soil. The proportion of cement alters the plasticity, volume change, susceptibility to frost heave, elastic properties, resistance to wet-dry and freeze-thaw alternation, and other properties in different degrees for different soil.

Comparative compressive strength data on cement-treated soil mixture prepared with low (0.17 percent), medium (0.48 percent) or high (0.92 percent) alkali type I cement indicated that high alkali content is beneficial to strength if the soil contains a relatively high proportion of clay-free quartz surfaces. (17)

Using type III cement on loamy sand, the 7 and 28 day dry compressive strengths were about 2 and 1.4 times the

Table 16

Cement Requirements of AASHO Soil Groups. (16)

AASHO Soil Group	Usual Range in Cement Requirement		Estimated Cement Content and that Used in Moisture-Density Test, Percent by Wt.	Cement Contents for Wet-Dry and Freeze-Thaw Tests, Percent by Wt.
	Percent by Vol.	Percent by Wt.		
A-1-a	5- 7	3- 5	5	3- 5- 7
A-1-b	7- 9	5- 8	6	4- 6- 8
A-2	7-10	5- 9	7	5- 7- 9
A-3	8-12	7-11	9	7- 9-11
A-4	8-12	7-12	10	8-10-12
A-5	8-12	8-13	10	8-10-12
A-6	10-14	9-15	12	10-12-14
A-7	10-14	10-16	13	11-13-15

strength of the values for type I cement respectively; for a silty clay loam the strength for type III was only slightly greater than for type I cement. (18)

Moisture Content. - Differences in water hardness does not cause significant differences in the quality of cement-treated soil. Sea water has been used successfully in construction. (19)

The more intimate the soil-cement-water mixture, the stronger and the more durable the resulting soil-cement will be.

The effect of moisture content on the quality of soil-cement largely arises from its influence on the compaction; for good compaction it is necessary to bring the material to

the maximum dry density with minimum effort. The highest compressive strength can be obtained with specimens compacted slightly below the optimum moisture content while the best results in durability test are obtained at moisture content somewhat above the optimum. (20)

Pulverization, Mixing and Compaction. - The quality of the cement-treated silty and clayey soils is highest when 100 percent of the soil, exclusive of gravel or stone, is pulverized to pass a No 4 sieve. However, the quality is not seriously affected by the presence of as much as 30 percent unpulverized soil provided the lumps are moist (at or slightly above optimum) at the time of compaction of the cement-treated soil. If the lumps are dry at the time of compaction the quality of the mixture may be seriously impaired. (18)

The efficiency of the mixing and compacting equipment and the time required for mixing and compacting influences both the strength and durability of cement-treated soil. Mixing involves both degree and time. The degree of mixing also termed "uniformity of mixing" or efficiency of mixing is a measure of thoroughness or completeness of mixing compared to some arbitrary standard. The degree of mixing has been measured experimentally either by direct measure of uniformity of cement concentration by means of radio-active tracer technique, or measured by the ratio of strength of specimens molded from the field mix to the strength of specimens molded from the field mix after additional laboratory mixing. British

engineers experience indicated that 60 percent efficiency is typical for mixed-in place work by multi-pass processes for cohesive soil. Higher efficiencies are obtained using the multi-pass process with granular soils and using the single-pass process with cohesive soils. Efficiencies approaching 100 percent are obtained using plant mix with granular soil.

Duration of mixing period; Increasing the period of moist mixing and/or delay in compaction following completion of moist mixing, generally increases the optimum moisture content, reduces the maximum density, decreases the compressive strength, and increases the brushing losses in the wet-dry and freeze-thaw tests. The degree of the influence on each varies widely depending on the soil type, the period of mixing or period of delay, and the cement content. Extended mixing reduces densities of sand in the order of 0 to 1 pcf of sandy loams up to 3 pcf, of silty soil 2 to 4 pcf and of clays up to 5 pcf or more. Prolonged delays without mixing may increase these even more. Maximum values recorded for extended mixing and delays range from 8 pcf (21) to 11 pcf (22).

To obtain satisfactory soil-cement adequate compaction is essential. This is shown by the fact that in the laboratory a decrease in dry density of 1 pcf has been found to cause a decrease in compressive strength of between 20 to 40 psi and a greater proportionate loss of durability. (20)

Curing. - The manner in which soil-cement is cured influences the resulting product. As with concrete the strength

of soil cement increases with age and like concrete, soil-cement must be kept moist during the initial stages of cure. In field curing, various types of bituminous material, moist soil, waterproof paper, and calcium chloride curing methods were used with good result.

Temperature during cure has a marked effect with higher temperature increasing the rate of curing. Soil-cement will harden at all temperatures above freezing, but warm-weather-cured soil-cement appears to be stronger at least during the first few months after preparation.

Admixtures. - Soil admixtures and additives have been used to improve the reaction between the soil and the cement since the earliest projects. Sand, clay, lime, bituminous emulsion, flyash and many different kinds of chemical additives have been studied and tested for different reactions. Lambe, Mohr, and Michaels (7) in a series of reports dated from 1957-1959 laboratory studies have shown that dramatic improvement in the strength of soil-cement can be obtained by the addition of small amounts of certain chemicals to reduce the amount of cement required and to stabilize some soils which are not responsive to cement alone, the following Table 17 shows the economic considerations of certain of these additives.

Table 17

Cost Comparison of Soil Stabilized with Cement Alone and with Cement Plus Additive for Required 7-Day Strength.

Soil	Cement Content, % by Dry Soil Weight	Additive	Additive Concentration % by Dry Soil Weight	Stabilizer and Additive Cost, Dollars per Cu. Yd. Stabilized Soil	Percent Saving in Material Cost by Use of Additive over Cement Alone
New	11.0	0	0	3.21	0
Hampshire	7.5	NaOH	0.9	3.18	1
Silt	6.5	Na_2CO_3	1.0	2.60	19
	5.0	Na_2SO_4	0.8	1.86	42

Cement-Modified Soil

Cement may also be used as an admixture with soil to change their undesirable characteristics and "modify the soil into a more favorable construction material".

Cement has been used to improve bearing values of granular base and subbase materials, to reduce their plasticity or swell characteristics, to prevent consolidation and pavement pumping at the joints (erosion), and to produce a firm working table as a subbase, (16) as shown in the following Tables 18, 19 and 20.

Table 18

Permanency of Plasticity Index Reduction of Cement-Modified Soil.

Type of Test	Cement Content, Percent by Vol.			
	0	1	3	5
Raw Soil	14	-	-	-
Lab mixture, age 7 days	--	5	4	NP
Lab mixture after 30 cycles freezing and thawing	--	8	3	NP
Lab mixture after 60 cycles freezing and thawing	--	9	1	NP

* A-2-6 Soil from Carroll County, Tennessee (16)

Table 19

Permanency of Bearing Values of Cement-Modified Soil.

	CBR
Raw soil (A-1-bco) disintegrated granite material from Riverside County, California	43
Lab mixture, age 7 days, 2% cement by weight	255
Lab mixture after 60 cycles of freezing and thawing, 2% cement by weight	258
Lab mixture age 7 days 4% cement by weight	485
Lab mixture after 60 cycles of freezing and thawing, 4% cement by weight	574

(16)

Laboratory research and field work showed that cement may be used most effectively to reduce the volume change characteristics and to increase the load-carrying capacity of silty clay subgrade soil. (16)

Table 20

A-7 Soil Plasticity Affected by Cement and Age

Test Condition	Plasticity Index
Raw soil A-7 clay from Comanche County, Oklahoma	28.5
Lab mixture, 7% cement by volume	14.5
Field mixture after construction, 7% cement by volume	10.5
Field mixture after 6 years of service, 7% cement by volume	5-11

(16)

Relatively small quantities of cement flocculate the fine soil grain, perhaps by a combination of base exchange phenomenon and cementing action, to form small conglomerate masses of new soil grains or aggregates. These new soil aggregates will have lowerplasticity and volume-change characteristics than the soil and greater load-bearing capacity over a wider range in moisture content. The degree of modification of the soil will vary within the amount of cement added, and therefore a cement-modified soil can be produced that will have the characteristics required in reducing sensitivity to

water and improving the load bearing capacity for the particular structure under consideration.

The first cement-modified clay soil project was an experimental treatment of subsoil under concrete pavement built in 1921 - 1922 by the California Highway Department between Denverton and Rio Vista. During the 30 years period under observation "there was considerable evidence that the cement was properly incorporated and there was no deterioration of the cement-soil mixture". (16)

The data in Table 18 shows examples of the permanency of cement modification.

Modulus of Elasticity and Poisson's Ratio of Soil-Cement

The static modulus of elasticity in compression was computed by the Portland Cement Association as the secant moduli at 33 percent of ultimate compressive strength of 2.8 in. diameter by 5.6 in. height specimen. This modulus increased with increasing cement content; average values at 28 days varied from about 100,000 psi to 2,000,000 psi for the cement-treated sandy soil mixture and from about 260,000 psi to 760,000 psi for the cement-treated silty soil mixtures. The modulus also increased with age. In some cases the 90 day values were double the 7 day values, but the increase averaged only about 50 percent of 7 day values. Dry cement-treated soil specimens had a lower modulus of elasticity; the decrease was slight for the sandy soil mixtures but was as much as 60 percent for the silty soil mixtures.

For the values of static modulus in flexure, the sandy soils showed values ranging from 800,000 psi to 4,300,000 psi, and values for the clayey and silty soils ranged from 700,000 psi to 3,500,000 psi (19).

Dynamic Poisson's ratio, determined from fundamental transverse and torsional frequencies of beams, range from 0.22 to 0.27 for granular soils; from 0.30 to 0.36 for clayey, and from 0.24 to 0.31 for silty soil, values of static Poisson's ratio determined from the ratio of lateral strain to axial strain in elastic range (between 10 and 40 percent of ultimate strength), exhibited a random variation with cement content and age, and averaged 0.14 for cement-treated sandy soil mixtures and 0.12 for the cement-treated silty soil mixtures. (22)

Stresses Under Footing

Elastic-Theory for Estimating Stress in Soil

The equations expressing the stress components caused by a perpendicular point load, to a unit surface within an elastic, isotropic, homogeneous mass which extends infinitely in all directions from a level surface, are attributed to Boussinesq. The equations:

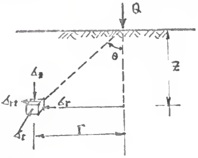


Fig. 22. Stresses in Cylindrical Coordinates Caused by a Surface, Vertical Load. (23)

$$\sigma z = \frac{Q}{2\pi Z^2} (3 \cos^5\theta)$$

$$= \frac{\theta}{Z^2} \frac{\frac{3}{2\pi}}{[1 + (\frac{r}{Z})^2]^{5/2}}$$

$$\sigma r = \frac{Q}{2\pi Z^2} [3 \sin^2\theta \cos^3\theta - \frac{(1 - 2\mu) \cos^2\theta}{1 + \cos\theta}]$$

$$\sigma t = \frac{Q}{2\pi Z^2} (1 - 2\mu)(\cos^3\theta - \frac{\cos^2\theta}{1 + \cos\theta})$$

$$\tau rz = \frac{Q}{2\pi Z^2} (3 \sin\theta \cos^4\theta) \qquad (23)$$

The coefficient designated by μ is Poisson's ratio, it has values in elastic material that are always between 0 and 0.5 and in soil it is assumed to be 0.5.

It has been found that estimates of settlements obtained by use of the Boussinesq equations for determination of stresses are usually larger than the observed settlement.

Typical clay strata usually have partings or thin lenses of coarser material within them, so this is a non-isotropic condition which causes a greatly increased resistance to lateral strain. In 1938 Westergaard assumed that an elastic material is laterally reinforced by numerous closely spaced, horizontal sheets of negligible thickness of infinite rigidity, which prevents the mass as a whole from undergoing lateral strain, an extreme case of non-isotropic condition. (24)

Westergaard's expression for the vertical stress caused by a point load is

$$\sigma z = \frac{Q}{z^2} \frac{\frac{I}{2\pi} \sqrt{\frac{1-2u}{2-2u}}}{[\frac{1-2u}{2-2u} - (\frac{r}{z})^2]^{3/2}}$$

u = Poisson's ratio

At points directly below the load the stresses have minimum values when Poisson's ratio has its minimum value of zero. This formula gives values of vertical stresses which are approximately equal to two thirds of the values given by the Boussinesq formulas and plot as a flat curve for a case of large lateral restraint. (23)

In the active zone under a foundation Terzaghi (11) pointed out, that the Dn, of (1/n) σo isobar increases in direct proportion to the width B of the loaded area. For practical purposes Terzaghi recommended the value of n = 5 for round or square footings, which means selecting an isobar characterized by 1/5 σo the vertical contact stress. The depth of such an isobar is then denoted by Dn = 5 as shown in Fig. 23. This recommendation was supported by Terzaghi's observation that direct stresses are considered of negligible magnitude when they are smaller than 20% of the applied stress.

If individual footings are spaced closely enough the individual isobar of an equal intensity of Dn = 5 = (1.5) B will intersect forming a single pressure bulb under the structure.

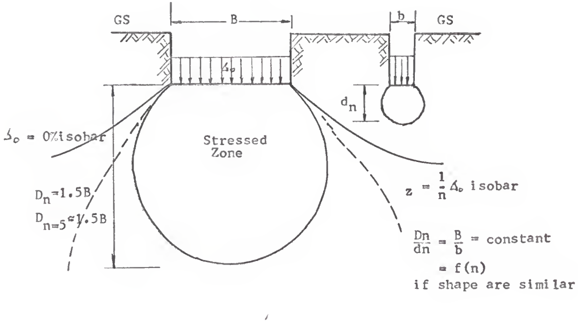


Fig. 23. Effect of Width of Footings on Depth of Isobars. (25)

Gillette (26) based on the original preliminary assumptions of Boussinesq for point loads, Love's differential equations for the deflections and stresses beneath uniformly loaded flexible circular surface foundation, and eliptic integral tables developed "influence values" for the convenience of practical foundation design problems.

The values of Poisson's ratio (u) does not affect the stresses acting vertically but does cause stress variations in all stresses acting at an angle with the vertical. Fig. 22 shows the values necessary to determine stresses under circular flexible foundation under the ideal assumed condition.

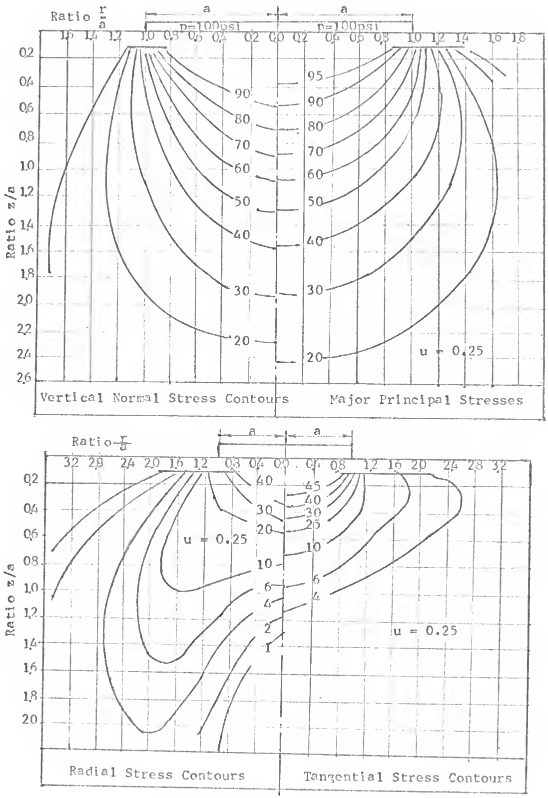
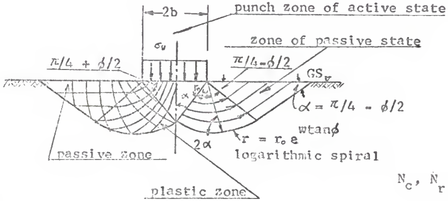


Fig. 24. Pressure Bulb Values Under Circular Flexible Foundation. (22)

Soil Bearing Capacity by Means of Plastic Equilibrium Theory

The elastic deformations with most common materials are extraordinarily small, whereas plastic deformations are comparatively large, and for the purpose of simplification the elastic deformations are fully ignored, and the elastic part of the body is treated as a rigid body. Within the elastic range the plastic deformations can be considered so small that the geometric shape of body would not be greatly altered. The volume changes, is also assumed to be zero in the plastic region of the body so that the remaining deformation can be construed as plain slidings. Upon this reasoning Prandtl in 1921 presented an expression for the ultimate bearing capacity of strip shaped loaded areas, as shown in Fig. 25.



α : rupture angle

γ : unit wt. of soil

r : initial radius of spiral curve.

r : radius of pt. on the sprial curve.

ϕ : internal friction angle.

z : depth of footing

c : cohesion of soil

N_c, N_r, N_q : bearing capacity factors based on Ø

Fig. 25. Prandtl's System of Study. (25)

$$qu = \left(\frac{c}{\tan\phi} - \frac{1}{2}\,\gamma b\sqrt{kp}\right)(Kpe^{\pi\tan\phi} - 1)$$

$$\text{where } Kp = \frac{1 + \sin\phi}{1 - \sin\phi}$$

Based on Prandtl's theory of plastic failure Terzaghi presented a modified system for a shallow strip footing ($Z = < 2b$) there ground surface at the base line of the footing is loaded with a uniformly distributed load, $q = \gamma z$.

$$Q_{crit} = 2bc\left(\frac{Kpc}{\cos^2\phi} + \tan\phi\right) - 2bq\,\frac{Kpq}{\cos^2\phi} + \gamma b^2 \tan\phi\left(\frac{Kpr}{\cos^2\phi} - 1\right) \qquad (25)$$

first term is effect of cohesion, Q_c.
second term is effect of surcharge, Q_q.
third term is effect of weight of wedge of active state zone, Q_r.

$$* \; Q_{crit} = Q_c + Q_q + Q_r = 2b\,(c\,N_c + \gamma z\,N_q + \gamma\,b\,N_r) \qquad (25)$$

Mixing and Compaction of Soil-Cement in Confined Area for Foundation Purpose.

Mixing

Usually the separation of the footings are great enough for passage of a conventional soil-cement construction machine with proper protection for the excavation. The two methods described below will allow working a small quantity of soil-cement for the foundation purpose.

A. After excavation by backhoe, excavator, or clamshell, which places the soil in a wind row along the side of footings (or strip footings), the cement can be spread and mixed by a rotary mixer by multiple-passes. After uniform mixing the soil-cement is replaced by a bulldozer and/or loader, and layer by layer, compacted by the vibro-compactor.

B. During excavation, the soil is stock-piled in a proper place, and pulverized if necessary. It is then mixed by a traveling mixer (paving mixer) and directly placed into the foundation trench by the paver and compacted to the desired density.

The second method can save the soil-cement lost during mixing and back filling thus gaining better control.

<u>Compaction in Confined Areas Such as a Trench</u>

This must be accomplished according to varying kinds of compaction machines of the capacity and dimension to be used in a confined area such as a structure footing trench with regard to machines available for the compaction purpose. The single-unit vibrating base-plate-type compactor appears to be the most satisfactory. The table below shows maximum dry unit weight and optimum contents from laboratory compaction test compared with values obtained in tests with single-unit vibrating base-plate-type compactors.

For single-unit, self-propelled, base-plate-type vibratory compactor, the width of the strip compacted varies from 1 ft

Table 21

The Compaction Capacity of Vibrating Base-Plate-Type Compactor.

Soil	Lab. Compaction				Data on Compactor and Its Operation						Field Compaction			
	Brit. Std.		Max. AASHO								Dry unit Wt.			
Type	Max Dry Unit Wt (pcf)	OMC (%)	Max Dry Unit Wt (pcf)	OMC (%)	Gross Wt. (lb)	Contact Area (sq.in)	Static Pres. (psi)	Freq. (cpm)	No. Cover-ages	Loose Lift Thick	Max (pcf)	% of Brit Std Max.	% of Mod. AASHO Max.	OMC
Heavy Clay	99	24	116	16	1,480	660	2.2	1,200	16	9-12	103	104	88	21
	99	24	116	16	1,570	570	2.75	1,500	16	9-12	87	87	75	20
	97	26	113	17	4,480	1,700	2.6	1,050	16	9	98	101	86	17
Sandy Clay	115	14	128	11	4,480	1,700	2.6	1,050	16	9	117	101	91	15
	109	16	126	12	1,480	660	2.2	1,200	16	9-12	116	106	92	16
	109	16	126	12	1,570	570	2.75	1,500	16	9-12	114	104	90	16
Graded Sand	121	11	130	9	530	280	1.9	1,800	10	9	128	105	98	10
	121	11	130	9	1,480	660	2.2	1,200	16	9-12	135	111	103	8

to 3 ft, ranges of working speed from 0.22 to 1.0 mph.

Another type compaction machine suitable for this purpose is the drop hammer compactor of either the explosion type or winch type. Cost for the compaction by this type of equipment would be greater than for the vibratory compactor.

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STABILIZATION OF LOESS BY CEMENT FOR FOUNDATION PURPOSES

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ABSTRACT

A study of soil stabilization of loess soils by cement and sodium sulfate is presented. The purpose of the study was to investigate the possibility of in-place stabilization of loess soils which would allow the use of spread footings on this weak soil as an economical solution of foundation design compared to the presently used pile foundations.

A review of the existing literature shows the basic failure mechanism of foundations in loess soils and was used as a guide for the selection of types and quantities of additives to be used in this study.

Results of laboratory research shows the loess soil from Kansas City, Kansas, reacted favorably with Type III Portland Cement to compressive loads but the addition of sodium sulfate caused unfavorable reactions as shown by the wet-dry tests.

It was concluded that cement stabilized loess soils can be used as a base for spread footings. It will distribute the structural loading over a greater area thus reducing the stress in the untreated soil and it is not affected by moisture variations which cause great settlements in the raw loess soils.